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Ultimate limit states design of concrete railway sleepers

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The railway industry has been slow to adopt limit states principles in the structural design of concrete sleepers for its tracks, despite the global take up of this form of design for almost every other type of structural element. Concrete sleeper design is still based on limiting stresses but is widely perceived by track engineers to lead to untapped reserves of strength in the sleepers. Limit design is a more rational philosophy, especially where it is based on the ultimate dynamic capacity of the concrete sleepers. The paper describes the development of equations and factors for a limit design methodology for concrete sleepers in flexure using a probabilistic evaluation of sleeper loading. The new method will also permit a cogent, defensible means of establishing the true capacity of the billions of concrete sleepers that are currently in-track around the world, leading to better utilisation of track infrastructure. The paper demonstrates how significant cost savings may be achieved by track owners.

Notation

f	frequency of occurrence of an event
I	incremental impact force
I_{eff}	ultimate or design incremental impact force
LF_i	ith load factor
L_i	ith type of load
M_I	moment induced by load I
M_Q	moment induced by load Q
M_u	ultimate bending strength
n	number of standard deviations
Q	transient non-impact load
R_u	ultimate strength
S^*	factored design load
V_i	coefficient of variation
δ_i	bias coefficient
φ	capacity reduction factor

1. Introduction

A railway track not only provides the support for trains to travel from one location to another, it is also a complex structural system in which many non-linear elements interact dynamically. An essential part of this system is the set of railway sleepers that support and constrain the steel rails and spread the large gravity and dynamic forces from the trains down into the stone ballast and soil beneath. The sleepers are elements that act primarily in bending in response to the two point-forces transmitted downwards from the rails, and to the distributed pressure upwards from the ballast. Nevertheless, sleepers have usually not been subjected to rigorous and rational design principles because visually graded timber was adequate for resisting the loads on sleepers in most countries, from the construction of the first railways 200 years ago until relatively recently.

In recent decades, however, the use of timber for sleepers has become viewed in many countries as an unsustainable choice because of diminishing availability and quality of source material. As a replacement, concrete sleepers have been widely adopted because of their stability, long life, resistance to biological attack and inherent strength. There are now tens of millions of prestressed and reinforced concrete railway sleepers manufactured and installed in rail track around the world every year. This is a huge capital investment by private and public track owners, measuring in the billions of dollars annually, so it is important that these sleepers are designed effectively and according to rational engineering principles. If not so designed, their capacity will not be known clearly and track owners could be wasting a significant amount of capital, representing an unacceptable economic loss to their operations.

Currently in many countries, concrete sleepers are designed according to a nineteenth century deterministic method called ‘permissible stress design’ (e.g. AS1085.14-2009 (Standards Australia, 2009a), AREMA (2010)) which was the reigning method for engineering until the 1970s. From that time, design codes around the world began to switch across to the limit states method after extensive research showed that use of limit states would lead to outcomes that were more effective and more economical (Melchers, 1987). Today virtually every structural code around the world uses limit design, except for codes used in the design of concrete railway sleepers; the inherent conservatism of permissible stress design helps explain the widespread belief among track engineers that concrete sleepers have unused reserves of strength – sleepers are generally replaced only because of non-design factors such as serious damage due to train derailment or inappropriate materials in the concrete mix or manufacturing faults.

The economic consequences of having sleepers rated lower than their true capacity are not insignificant. For example, when a track owner purchases new sleepers, the owner is paying for components that can probably support heavier trains with more passengers or freight than the track is rated for; the owner is therefore denied the extra fees that would have been paid by operators of heavier trains. In track with existing sleepers, if the owner wants to allow access to trains that are heavier than the supposed capacity of the sleepers, those sleepers would have to be removed and then replaced with new higher capacity units, all at a very great and unnecessary cost. There is clearly a need for a method of design and rating of concrete sleepers that is more rational than permissible stress design and which allows for the inherent stochastic variability of element strengths and of applied loads.

There has been a little work towards a limit state or probabilistic-based method of design for sleepers. Murray and Cai (1998) proposed this method as a way to ensure concrete sleepers were being utilised effectively. Wakui and Okuda (1999) concentrated on analysing the forces applied to sleepers as a preliminary investigation towards a limit state design approach. Lilja *et al.* (2008) developed a finite-element model for a complex investigation of the optimisation of stresses in sleepers considering the stochastic variations in factors such as ballast support.

Over the past few years a number of research projects in Australia have worked towards the goal of limit state design. For example, Leong (2007) studied the dynamic response of sleepers and undertook a unique set of measurements of the range of static and dynamic forces that are applied to a heavy-haul railway track with a view to developing a limit state methodology. Leong identified the inadequacy of the permissible stress approach that ignores the range of magnitude and frequency of the forces applied to sleepers. Grassie (1995) discussed how the range of force frequencies of interest for damage to sleepers can be as high as 1500 Hz, whereas Nielsen (2008) described wheel–rail impact forces with frequencies up to 2000 Hz.

Remennikov *et al.* (2007) described dynamic laboratory testing of sleepers as well as analysis of track forces, establishing some relationships needed before Australia's concrete sleeper design code can be converted to limit state design. Murray and Leong (2009) developed a method of categorising track into levels of importance to aid in determining suitable probabilistic design forces, and proposed a series of limit states by which sleepers could be designed. More recently Kaewunruen *et al.* (2011) described the need for a limit states design methodology and suggested potential savings in sleeper manufacture by use of such an approach. Nairn and Stevens (2010) presented a probabilistic method of design for low-profile concrete sleepers based on extensive measurements of sleeper behaviour especially under low-stress high-cycle fatigue conditions.

Nevertheless, none of this research has until now produced a limit state design equation for concrete sleepers with values of partial load factors and capacity reduction factors.

The authors believe that the equations and factors proposed in this paper constitute the first cogent limit design methodology that has been proposed for concrete sleepers. Importantly, the methodology utilises a probabilistic evaluation of the high-level impact forces that are responsible for bringing a sleeper to the ultimate limit state of failure. The paper does not explore limit states other than ultimate strength because these are still in the process of development. That work is building upon the proposal by Murray and Leong (2009) that there should be three limit states for concrete sleepers, namely strength, serviceability and fatigue. They described how serviceability of sleepers is related more to the development of a cluster of failed sleepers rather than deflection or vibration of an individual sleeper, and how the fatigue limit is better considered as a measure of the damage progressively accumulated by a sleeper. Because the proposed serviceability and fatigue limit states are closely related to sleeper failure, the development of the strength limit state is the first priority.

2. Limit design and in-track loads

The parameters in limit design that need to be addressed to ensure satisfactory margins of safety for the elements in-service, are illustrated in the well-known limit state of strength design equation

$$\varphi R_{\mathrm{u}} \geqslant S^{*} = \sum (LF_i * L_i) \tag{1}$$

where φ is the capacity reduction factor applied to the ultimate strength R_{u} of the element, $S*$ is the factored design load applied to the element, LF_i is the ith load factor applied to L_i the ith type of load. To apply Equation 1 to the design of sleepers, one must determine the parameters on the strength side of the equation, namely R_{u} and the associated φ reduction factor, and on the right side of the equation one must determine the type and magnitude of the loads sustained by the sleeper and the factors to be applied to those loads.

2.1 In-track loads

A railway sleeper experiences a spectrum of forces that is quite different from that experienced by elements in buildings, bridges and the like. In the existing permissible stress method nominal track loads are used, but in limit state design the actual spectrum of forces is needed and so in-field measurements are required. For this purpose, and as part of a larger ongoing project studying the design, capacity and life of concrete sleepers at Queensland University of Technology (QUT), a section of a commercial, heavy-haul, narrow-gauge railway track in Central Queensland, Australia, was selected for installation of a device known as a wheel impact detector. Full details of those track force measurements and how they were obtained from the detector are provided in Leong and Murray (2008), but in summary nearly 3 million readings were taken of the forces applied by train wheels to the rails over a period of 12 months. Some of the results of those in-field tests are described below.

(*a*) Gravity loads. For a railway sleeper, the permanent load it sustains is its own self-weight and the weight of the rails it supports, but that is only about 0·3% of the weight of a loaded wagon running over the track, and so is not significant. The more important gravity load carried by sleepers is the weight force applied as each individual axle passes quickly over a sleeper. Although these are gravity loads they should be thought of as multiple transient forces that are usually called 'quasi-static' loads in railway parlance. The distribution of the wagon gravity loads per axle measured at the test site is shown in Figure 1. In the trains passing over the test site there were four axles per wagon, about 100 wagons per train and 10 fully loaded trains per day, giving about 1·5 million transient loads from full wagons applied to a sleeper per year in Figure 1. The nominal or design axle load specified by the track owner for this rail line is shown in Figure 1 as 28 t (2 × 137 kN) but the graph shows that the maximum axle loads for full wagons were well above 28 t. Converting Figure 1 to kiloNewtons, the distribution is approximately normal with a mean force at one wheel-rail interface of 128 kN and standard deviation of 13 kN.

(*b*) Dynamic loads. When the steel wheels on a train become worn asymmetrically they can become 'out-of-round' and can even develop small flat spots on the wheel tread. These imperfections strike the head of the rail each time the wheel rotates and if they are severe enough they can generate very large impact forces to the head of the rail which get transmitted down into a sleeper beneath. These impact forces are quite different from the gravity forces that are caused by the vehicle mass; impact forces were measured at the test site which were just as large from empty wagons as from full ones. The wheel impact detector at the test site measured these forces, and the distribution of wheel impact forces over the 12 month period of testing is shown in Figure 2. These data show only the dynamic force over and above the static wheel load, that is, the incremental impact force, and only those impact forces of significance, defined as impact forces larger than 50 kN. The diagram is derived from data presented in Leong (2007) but adjusted to allow for the fact that the detector measured the impact forces occurring over a distance along the rail head of 3 m or more, but a given sleeper experiences full impact only if the impact occurs on the rail head above that sleeper. The distribution in Figure 2 is not of a normal shape but is heavily skewed like many natural stochastic processes such as wind speeds, rainfall and earthquakes. A Weibull distribution fits Figure 2 best, as portrayed in Equation 2

$$f = \alpha/\beta^{\alpha}(I-\delta)^{(\alpha-1)}\mathrm{e}^{-((I-\delta)/\beta)\exp(\alpha)} \quad (2)$$

where f is the normalised frequency of occurrence per year of

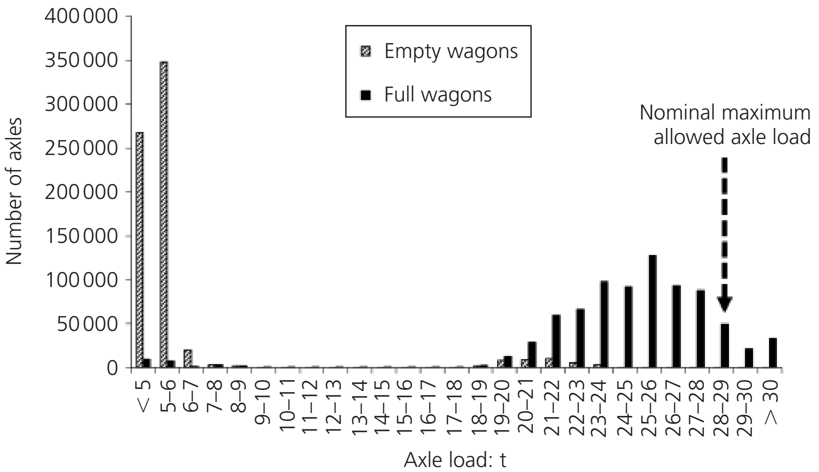

Figure 1. Distribution of wagon weight transmitted through axles, 2005–2006

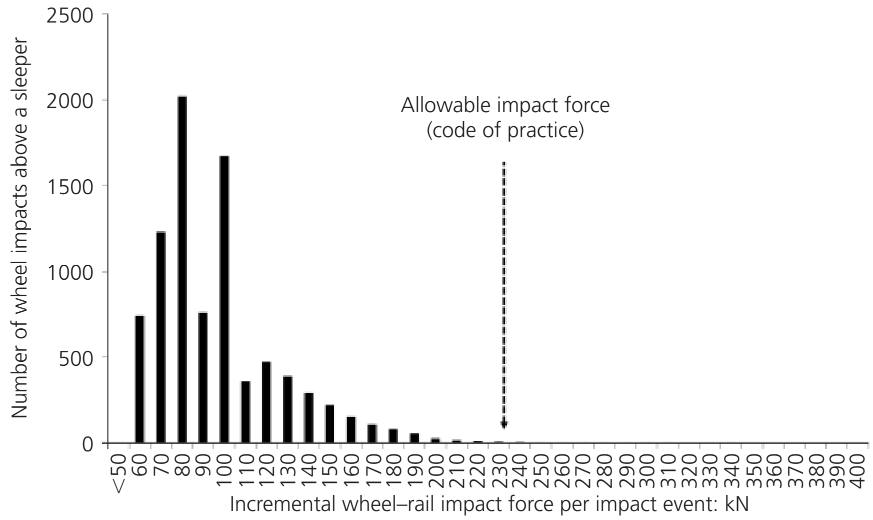

Figure 2. Distribution of incremental impact force transmitted through wheels, all wagons 2005–2006

a wheel–rail impact force of magnitude I in kN at a location on the rail directly above a sleeper, $\alpha = 1{\cdot}25$, $\beta = 39$, $\delta = 59{\cdot}5$; to obtain the actual number of impacts per year per sleeper, f should be multiplied by 129 000. This relationship, together with the normal distribution parameters from the gravity loads earlier, is needed in order to establish the load factors for limit design in Equation 1.

3. Analysis of loads for load factors

3.1 Load factors

From the discussion above there are just two types of load to be considered in the limit state of strength condition for sleepers, namely the transient weight force from the passage of individual axles of the train and the impact force applied by out-of-round wheels; the magnitude and frequency content of these two types of force act independently of each other because they have different origins. The limit state of strength Equation 1 can therefore be rewritten as

$$\varphi R_{\mathrm{u}} \geqslant S^{*} = LF_{\mathrm{w}} * Q + LF_{\mathrm{I}} * I_{\mathrm{eff}} \tag{3}$$

where LF_{w} is the load factor and Q is the nominal or design force for the transient weight component of load; and LF_{I} is the load factor and I_{eff} is the nominal or design force for the impact component of the load. Q and I_{eff} are derived from an analysis of the probability distributions of the two types of load as will be described later. Both parameters represent the values of wheel–rail force applied to rail above an individual sleeper and are determined from a consideration of the response of the track to the impulse forces applied at the wheel–rail interface.

The overarching condition that sets the magnitude of the load factors in limit design is the need to ensure an adequate factor of safety against failure during normal service operation. The parameter widely used to define safety and level of risk in systems is the reliability index β. Australian standard AS5104-2005 (Standards Australia, 2005) provides principles for setting levels of reliability for structures and because concrete sleepers are a major structural element in the track supporting system, these principles can be applied to them. The standard suggests that for ultimate limit states design in which Weibull and Gumbel distributions are characteristic of the actions and element resistances, β should be between 3·1 and 4·3, depending on the consequences of failure.

Concrete sleepers are most often used in tracks that require a high degree of reliability of operation. Such tracks include high-speed passenger lines where derailment of a train travelling at 300 km/h would be catastrophic, and lines that generate revenues which are critical to the business. The track described earlier which was used for the year-long measurement of wheel–rail forces is a heavy-haul, revenue-critical coal line. Failure of a single sleeper on a line of such high importance could result in significant speed restrictions and some interference with train operation, which would be classed as ‘moderate’ consequences; the failure of a group of sleepers, although much less likely, could lead to derailment which would be classified as ‘great’ consequences. From AS5104-2005 this combination of consequences sets the value of $\beta = 4{\cdot}0$, which is consistent with other structures that have consequences of failure between moderate and great, such as individual concrete bridge girders described by MacGregor *et al.* (1997).

3.2 Weight component load factor

Nowak and Lind (1979) in deriving load factors for bridges defined the load factor as

$$LF_i = \delta_i(1 + nV_i) \tag{4}$$

where LF_i is the load factor, δ_i is known as the bias coefficient, n is a measure of the level of confidence needed in the load factor and V_i is the coefficient of variation obtained by dividing the standard deviation of a distribution by its mean. Considering the transient weight forces of full wagons given in Figure 1, the mean and deviation of these forces gives $V_i = 0{\cdot}10$. The bias coefficient is simply the mean of the distribution of a parameter divided by the assumed or nominal value of that parameter, and so is a measure of the difference between these two values of the parameter. The mean and nominal values of the gravity weight loads of the wagons shown in Figure 1 were given earlier, giving $\delta_i = 0{\cdot}90$. The term n is simply the number of standard deviations from the mean, and Nowak and Lind (1979) suggest that n should be between 1·8 and 2·1, which correspond to points on a normal distribution that represent the upper 3·6th percentile and 1·8th percentile of the area under the curve. The upper 2 percentile point represents the chance of an event occurring 1 in 50 times, and represents a suitable characteristic value for the distribution of loads from wagon weights shown in Figure 1; for this case therefore $n = 2{\cdot}05$. Substituting these values into Equation 4 gives the load factor for the weight forces from the wagons as $LF_w = 1{\cdot}1$.

3.3 Impact component load factor

Because the distribution of impact forces in Figure 2 is Weibull-like, the method shown in Equation 4 for calculating load factor is inappropriate – the terms V_i and n are not able to represent a Weibull distribution adequately. Distributions such as Figure 2 are more appropriately considered as extreme event distributions in which the lower limit of the curve is zero but the upper tail trails off into infinity. The task then is to define a suitable upper limit that has a probability of occurrence appropriate to the importance of the element and to the loads being applied to the element. This approach is the same as that used for determining the design values of wind speeds and earthquake forces used in limit state of strength design of building infrastructure; such cases require determination of a suitable return period, otherwise known as the risk of annual exceedance, which in turn sets the magnitude of that event.

The Building Code of Australia (BCA, 2009) divides the importance of infrastructure into four categories with corresponding event return periods. The lowest importance is category 1 for temporary infrastructure with a return period of 50 years; the highest importance is category 4 for infrastructure which must survive a disaster, with a return period of 2000 years. As part of the larger on-going study of concrete sleepers at QUT, Murray and Leong (2009) drew on the approach of the BCA and proposed three categories of importance for railway tracks. Category 3, the highest importance, was proposed to be for 'lines that are critically important in terms of safety, revenue or business reputation' and would have a corresponding return period the same as the highest category of importance for buildings, namely 2000 years; that is, the annual probability of exceedance of the selected design wheel–rail force should be 1 in 2000 or 0·0005. For a concrete sleeper with a nominal life span of 50 years, this design force would therefore have a probability of occurrence of 1 in 40, or 2·5%, during the life span of the sleeper – that probability is similar to the 1 in 50 set earlier for the characteristic load appropriate to wagon gross weights.

So, in Equation 2 the incremental impact force I that would occur with an average annual frequency of occurrence of $f = 0{\cdot}0005$ at the wheel–rail interface above a given sleeper is $I = I_{\text{eff}} = 425$ kN, where I_{eff} is considered the 'ultimate' or extreme value of the event. Because this is an extreme event form of analysis, of necessity the load factor for the ultimate impact force is $LF_I = 1$ in Equation 3.

4. Capacity reduction factor

The only remaining term in the limit design Equation 3 yet to be determined is the capacity reduction factor φ. In structural codes such as AS3600-2009 (Standards Australia, 2009b) there are different φ factors for different types of action response in an element; for example, AS3600-2009 specifies $\varphi = 0{\cdot}8$ for elements subjected to bending, $\varphi = 0{\cdot}7$ for shear, and $\varphi = 0{\cdot}6$ for axial compression. Considering only bending, the question arises whether the AS3600-2009 value of $\varphi = 0{\cdot}8$ is appropriate for concrete sleepers; that is, would this value ensure the necessary level of safety established earlier ($\beta = 4{\cdot}0$) for the bending capacity of sleepers in track?

To investigate the effect of various values of φ on the reliability index, the structural reliability analysis program COMREL (2009) was employed. COMREL enables one to calculate the reliability index for systems in which one knows the characteristics of the distributions of the applied load and of the element's strength. The shapes and defining parameters of the distributions of the weight force of the wagons and of the incremental impact force were deduced earlier.

In order to determine the likely shape of the statistical spread of bending strengths of concrete sleepers, a Monte Carlo simulation was conducted using the cross-section dimensions and steel prestressing in a 'heavy' concrete sleeper typical for lines such as the test section of track described earlier. In the 1000 simulations conducted, each of the components and materials in the sleeper was varied randomly within bounds suggested by Kaewunruen and Remennikov (2008). The result is shown in Figure 3, which demonstrates sleeper bending strengths distributed normally about a mean of 54·2 kNm with a standard deviation of 3·3 kNm, giving a typical lower 5 percentile characteristic bending strength of

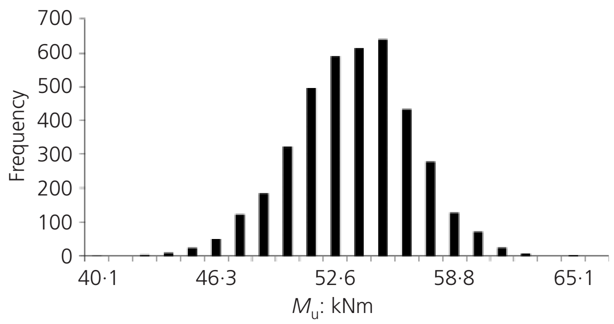

Figure 3. Distribution of concrete sleeper bending strengths with random variations in parameters

$M_u = 49{\cdot}0$ kNm. For $\varphi = 0{\cdot}8$, the corresponding maximum M^* would therefore be 39·2 kNm, of which 60% would be contributed by the ultimate impact force and 40% by factored wagon weight according to the values of I_{eff} and Q determined earlier for the traffic on the test track. These values together with the characteristics of their distributions were presented to COMREL and produced the relationship between φ and reliability index shown in Figure 4.

The value of φ corresponding to a reliability index of 4·0 is $\varphi = 0{\cdot}87$. A value of $\varphi = 0{\cdot}8$ corresponds to a reliability index of 4·3, so adopting the value of 0·8 from AS3600-2009 (Standards Australia, 2009b), which applies to all other concrete elements in bending, would not be unreasonable.

In summary, therefore, the limit state equation for designing concrete railway sleepers for the heavy-haul lines at the test site would be as follows, with the load factor rounded to the nearest decimal place as is typical in design codes

$$0{\cdot}8M_u \geqslant M^* = 1{\cdot}1M_Q + M_I \tag{5}$$

where M_Q in this case is the moment induced in the sleeper by

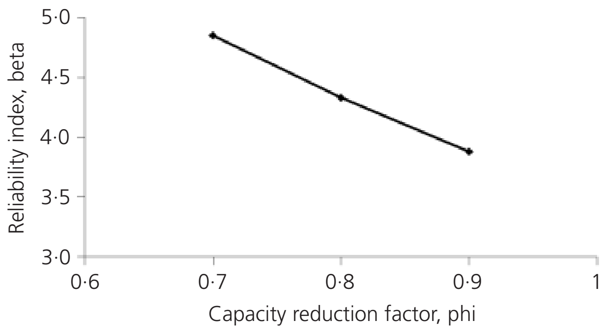

Figure 4. Relationship between reliability index β and reduction factor φ

the nominal or design value of the transient wagon weight force and M_I is the moment induced in the sleeper by the ultimate design value of the incremental wheel–rail impact force I_{eff} for the return period appropriate for the importance category of the track.

However, the derivation of M_I from I_{eff} is somewhat problematic. Murray and Leong (2009) showed how the relationship between wheel–rail impact forces and the response of the sleepers is very complex and can vary significantly depending on the dynamic characteristics of the many components making up both the track and the train. Around the world there are a number of sophisticated computer models of track and train, which could be used to determine M_I in a given track. However, Steffens and Murray (2005) in an international benchmarking exercise showed that the outputs of six of the most widely used models differed widely in many ways from each other and even from measured track data. Consequently, in the absence of a proven, reliable and workable methodology for relating wheel–rail force and sleeper moments that is applicable to limit states design at this time, the simplistic but widely used AS1085.14-2009 approach (Standards Australia, 2009a) has been adopted in this paper.

Furthermore, the distribution of ballast pressure under a sleeper has been assumed in this paper to produce maximum positive moments at the rail seat, but inadequate maintenance, ballast degradation and normal statistical variations in ballast density and grading can produce quite different distributions of pressure under a sleeper. Within the broader sleeper research programme at QUT a project is about to investigate the effect of these random variations on the propagation of forces through the track structure, and on the consequent bending moments and shear forces in concrete sleepers in the track.

5. Application of LSD equation

Track owners are often challenged by train operators with proposals for trains to carry ever greater freight or mineral volumes that in turn mean heavier wagons in the train, and so larger loads transmitted through the wagon axles to the track and to the sleepers.

To demonstrate the application of the LSD approach and the potential savings it can produce, a case study will be examined consisting of 100 km of broad-gauge track with sleepers designed for a nominal maximum axle load of 25 t, but it is proposed to run trains with 28 t axle loads on the track. The owner must decide if the existing sleepers can carry the extra load or will need to be replaced with 28 t rated units; this is a decision with major economic consequences. To purchase and install concrete sleepers in-track, the cost is more than Australian$200 per unit, so in 100 km of single track with sleepers at 685 mm centres, the outlay for the owner would be at least Australian$30 million. Clearly, being able to leave the existing sleepers in-track would result in substantial savings for the owner.

Broad-gauge sleepers have a length of 2·7 m and a rail-centre distance of 1·68 m, so the moments generated in these sleepers at the rail seat (i.e. at the point in the sleeper beneath the rail) can be calculated for 28 t axle loads with the same gravity and impact loads as described for the test section of the Queensland heavy-haul line earlier. The method for calculating rail seat moments is drawn from AS1085.14-2009 (Standards Australia, 2009a), assuming sleepers are spaced at 680 mm centres along the track.

$$M_Q = 0{\cdot}55 \times 137 \text{ kN} \times (2{\cdot}7 \text{ m} - 1{\cdot}68 \text{ m})/8 \\ = 9{\cdot}6 \text{ kNm}$$

$$M_I = 0{\cdot}55 \times 425 \text{ kN} \times (2{\cdot}7 \text{ m} - 1{\cdot}68 \text{ m})/8 \\ = 29{\cdot}8 \text{ kNm}$$

$$0{\cdot}8M_\mathrm{u} \geqslant M^* = 1{\cdot}1 \times 9{\cdot}6 + 29{\cdot}8 = 40{\cdot}4 \text{ kNm}$$

so the required $M_\mathrm{u} = 50{\cdot}5$ kNm for 28 t axle load.

Kaewunruen and Remennikov (2008) provided details of a broad-gauge concrete sleeper rated at 25 t axle load capacity but designed according to the old permissible stress principles. The dimensions of the sleeper adopted in that study are shown in Figure 5. Calculating the limit state of strength of a prestressed element using the approach in AS3600-2009 (Standards Australia, 2009b), this broad-gauge sleeper has an ultimate bending strength of: actual $M_\mathrm{u} = 52{\cdot}0$ kNm.

This bending capacity is 3% more than the required capacity of 50·5 kNm, which is adequate. However, the actual strength of 52·0 kNm is calculated on a nominal 28 day concrete strength of 50 MPa, but as described in Warner *et al.* (1998) the strength of concrete can increase by up to 30% in its first year, so it is likely that, as these sleepers age in-track, their capacity could well rise above even 28 t axle load. On the basis of this limited evaluation, the track owner might well allow the heavier trains to run on the track, with the increased revenue they would bring, and without the very large expense of re-sleeper-ing the track. Of course, before the heavier trains were allowed access to the track, the following additional conditions would need to be checked

(*a*) other forms of failure of sleepers such as shear, serviceability, fatigue
(*b*) so-called centre-binding causing large negative moments at the centre of the sleeper
(*c*) the increased stresses and actions in other track elements caused by the higher axle load, including the normal and octahedral shear stresses in the steel rails at the wheel–rail contact point, the bending and shear stresses in the rails, the pressure applied by the sleeper to the ballast, and the pressure applied by the ballast to the underlying soil.

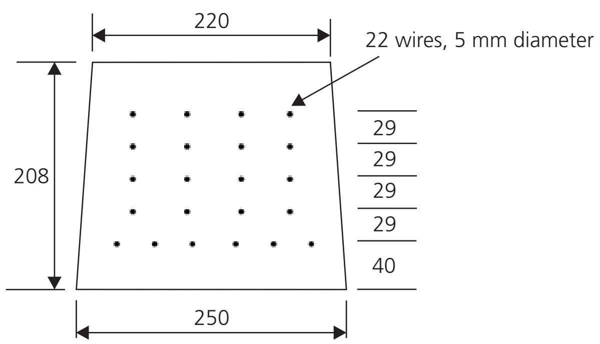

Figure 5. Cross-section dimensions of broad-gauge sleeper in case study

6. Limitations

There are other issues yet to be explored with the limit state method before it can be adopted as a standard for sleeper design and rating. Equation 5 is limited to heavy-haul lines similar to that used in this study – it is not yet known how the load and reduction factors and the ultimate impact force would change for different traffic types (e.g. freight and grain traffic) and different lines (e.g. passenger and mixed use).

As mentioned earlier, Murray and Leong (2009) discussed three limit states for concrete sleepers, namely strength, serviceability and fatigue, but only bending strength has been examined in the present paper. The other limit states require substantially more investigation.

Finally, as discussed herein, the relationship between the very short impulse force from wheel impact on rails and the consequent bending moments in concrete sleepers has not been reliably determined for general design use. The widely used quasi-static relationship in AS1085.14-2009 (Standards Australia, 2009a) has been used in this paper but the true relationship is far more complex and a workable form of that relationship is being developed as part of the wider research programme into the design and lifespan of concrete sleepers at QUT.

7. Conclusions

A long-term investigation is being conducted at QUT in Australia into the design, maintenance and lifespan of prestressed concrete railway sleepers. A part of that work in which a limit state equation was developed for establishing the design bending capacity of these sleepers is reported herein. The following conclusions may be drawn from the paper.

(*a*) Although concrete sleepers are currently designed using the old permissible stress design approach, they are important elements in the load-carrying function of railway track, so upgrading their design methodology to limit state is entirely appropriate.
(*b*) An analysis of the distributions of loads applied by trains to tracks shows that there are two main types of wheel–rail forces: a normal distribution of transient forces derived from the weight of train vehicles through individual axles; and a Weibull distribution of impact forces generated by imperfections in wheels and rails.
(*c*) The established system of rating the importance of infrastructure for determining ultimate forces due to extreme events such as wind and earthquake was adapted for rating the importance of railway tracks and deriving the ultimate wheel–rail forces for limit design.
(*d*) The well-established approach for deriving partial load factors for bridges and other structures was applied successfully to railway sleepers.
(*e*) A limit design equation was derived with load and capacity reduction factors set in accordance with appropriate reliability and safety considerations.
(*f*) Application of the equation of limit state of strength to a scenario for upgrading an existing track showed how this approach may provide very large savings for track owners, subject to consideration of all forms of sleeper failure and of the other limit state conditions described in this paper.
(*g*) The development of an analytically sound limit design equation together with its partial load factors is a first for railway sleepers. However, a significant amount of work remains to confirm the application of this approach to sleepers in-track, as well as to derive the methodology for all the proposed limit states for sleepers, and establishment of a workable and reliable methodology for determining the forces applied to and moments within sleepers under high-impact dynamic conditions that is readily applicable in limit states design.

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